Applied Geotechnical Engineering
& Geologic Consulting

August 31, 2013

Anderson Engineering of Minnesota, LLC
Attention: Gary Johnson, P.E.
13605 1st Avenue North, Suite 100
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SUBJECT: **Geotechnical Investigation**
Phase 1 of the Proposed Expansion
Eagle Point National Cemetery
Eagle Point, Oregon

At your request, Applied Geotechnical Engineering and Geologic Consulting LLC (AGEGC) has conducted a geotechnical investigation for the proposed Phase 1 section of the proposed expansion of the Eagle Point National Cemetery in Eagle Point, Oregon. The purpose of the investigation was to evaluate site conditions with respect to the proposed development plans and develop guidelines and criteria for construction of the new roadways, utilities and foundation recommendations. Our investigation consisted of a ground-level reconnaissance, subsurface explorations, limited laboratory testing, and engineering analyses. This report summarizes our findings and presents our geotechnical recommendations for the proposed Phase 1 expansion.

SITE DESCRIPTION

Phase 1 of the proposed expansion area consists of approximately 8 acres of undeveloped cemetery property located on the west side of Riley Road, east of the currently developed portion of the cemetery. The site is generally brushy with areas of scrub oak woods and grassy areas. The site typically is gently sloping down to the west from Riley Road. The western portion of the site is relatively flat. The west side of the site is bounded by single-family residences. Several of the residences have retaining walls along the cemetery property line.

PROJECT DESCRIPTION

Phase 1 of the expansion project includes site infrastructure consisting of utilities, the road system, retaining walls and entrance gates. In addition, Phase 1 facilities in this portion of the property include crypts and columbaria.

SUBSURFACE EXPLORATIONS

General. On October 3, 2012, fourteen test pits were completed for evaluation of subsurface conditions for the expansion of the Eagle Point National Cemetery. The test pit excavations were completed using a CAT 312 trackhoe equipped with a 3 ft rock bucket and provided and operated by Frank Norris

Excavation. On April 22 and 23, 2013, twelve soil borings were completed for the Phase 1 area at locations designated by Anderson Engineering. The soil borings were completed using an ATV-mounted drill rig provided and operated by Lawrence and Associates of Redding, California. Disturbed soil samples were taken periodically using the Standard Penetration Test (SPT).

The locations of the test pit and soil boring explorations were staked in the field by Anderson Engineering. The subsurface explorations were observed by a licensed geotechnical engineer/geologist from our firm who maintained a detailed log of the conditions and materials encountered. Representative soil samples were collected and stored in air-tight containers for transfer to our laboratory.

The test pit and soil boring logs are provided at the end of this report. The terms used to describe the soil and rock materials are provided in Tables 1A and 2A in Appendix A.

The subsurface explorations for the Phase 1 portion of the site encountered a variable thickness of clayey silt soils at the ground surface, typically between 2 and 5.5 ft thick in the explorations. In previous work for the cemetery, clayey silt soils up to 7 ft thick were encountered. The clayey silt is black to brown at the ground surface. The clayey silt is highly expansive (has a high shrink-swell potential) with expansive index between 60 and 100% (ASTM D 4829). Soils with a high shrink-swell potential have significant volume changes with corresponding changes in the soil's moisture content and can cause significant damage to foundations and concrete flatwork. The clayey silt soil is typically medium stiff to stiff; however, the relative stiffness can vary greatly depending on the natural moisture content at the time of sampling.

The surficial silt soils are typically underlain by soft to medium hard (RH-1 to RH-2) sandstone with local beds of siltstone and claystone. The sandstone grades from the consistency of a hard soil to hard rock (RH-3) with depth. The top of hard rock can vary significantly over relatively short distances. Most of the subsurface explorations encountered practical refusal on medium hard/hard sandstone (RH-2/RH-3) sandstone at depths ranging from 0.5 to 12 ft. The weathering of the sandstone decreases and the hardness of the sandstone increases with depth.

CONCLUSIONS AND RECOMMENDATIONS

Based on the results of this investigation and our experience with similar projects, it is our opinion that the site, from a geotechnical standpoint, the Phase 1 is suitable for the proposed improvements. In our opinion, the most important geotechnical considerations associated with the proposed Phase 1 improvements are the presence of expansive soils, locally shallow sandstone, the cut slopes and retaining walls along the residential properties at the western property line, and locally shallow (seasonal) groundwater.

Our conclusions and recommendations concerning site preparation and earthwork, foundations, retaining walls, roads, and utilities are summarized below.

Site Preparation. In our opinion, clayey silt soils are not suitable for support of the proposed roadways, concrete flatwork, buildings foundations, or any other structure that is sensitive to differential movements. The clayey silt soils have a high shrink-swell index and are highly expansive. Highly expansive soils have a significant change in volume with corresponding change in the natural moisture content. Changes in volume can cause significant differential movement in structure and associated damage.

The unsuitable soils should be overexcavated from within a horizontal distance of 3 ft beyond the edge of any structure. We recommend that the clayey silt soils be overexcavated to a minimum depth of 3 ft below paved areas, concrete flatwork, and foundations. Locally deeper overexcavation may be required in areas of deeper clayey silt soils, especially during periods of dry weather when the clayey silt soils are significantly dry of optimum.

The expansive soils are relatively weak and have a low shear strength. We recommend that the expansive soils be removed from all fill areas prior to placement of any structural fill.

If the clayey silt soils are to be used as structural fill or in landscaping areas, surficial organics should be stripped. We anticipate that stripping to a depth of about 9 in. will be required in most areas. Locally deeper overexcavation will be required to remove tree stumps, roots greater than 1 in, in diameter and pockets of old fill. The site strippings and wood debris are not suitable for use as structural fill and should be removed from the site.

Subgrade in areas of overexcavation must be protected from disturbance due to construction activities and climate (wetting, drying, and/or freezing). The subgrade should be evaluated by the project geotechnical engineer prior to placement of structural fill on the native subgrade.

We recommend that the overexcavation of unsuitable soils be completed using a trackhoe equipped with a smooth-lip bucket. The overexcavation of the unsuitable soils and placement of the structural fill should be completed as one continuous operation (the subgrade should not be allowed to dry during earthwork operations). The subgrade should be covered as the overexcavation is completed.

Final slopes should be graded no steeper than 2H:1V. Fill slopes should be overbuilt a minimum of 2 ft then trimmed back to final grades. The toe of the fill should be keyed in to bear on the rock. We recommend a minimum keyway of 12 ft wide along the toe of any fill slope. All expansive soils must be removed from the keyway area and replaced with structural fill.

There are existing retaining walls along the property lines at the north end of the site, adjacent to the residential properties. Several of the fences appear to be failing and/or moving downslope due to soil creep. To reduce the risk of new structures or fill impacting the existing residential retaining walls, new fills and structures should be founded at least 15 ft from existing walls and should be constructed on the underlying rock. We also recommend that the fences be inspected prior to construction and existing condition of the retaining walls documented.

Structural Fills. Fill for the project can consist of imported granular fill or treated on-site soils. Existing fill and expansive soils must be removed from all fill areas prior to placement of any structural fill.

Structural fill for mass grading of the site for roadways can consist of imported hard, angular (crushed) rock up to about 4 in. in size.

All fill placed for the building pads and concrete flatwork should consist of imported granular fill. The fill should be compacted to at least 95% of the maximum dry density as determined by ASTM D 698. As discussed above, the clayey silt soils are not suitable for use as structural fill. Structural fill for mass grading of the buildings building pads and concrete flatwork should consist of imported hard, angular (crushed) rock up to about 4 in. in size. The crushed rock for mass grading should have less than 12% passing the no. 200 sieve (washed analyses). Structural fill for fine-grading of the buildings building pads and under concrete flatwork can consist of crushed rock up to about 1 in. in size, such a ¾-in.-minus crushed rock. The crushed rock for fine-grading of the building pad should have less than 10% passing the no. 200 sieve (washed analyses).

As an alternative to imported granular fill, the on-site clayey silt soils with admixtures may be used as structural fill if properly placed and compacted. For clayey soils such as the ones at this site, we recommend an admixture of quick lime (CaO). The lime reacts with the clays and alters them, resulting in a non-expansive soil. The strength of the soils with the lime admixture increase over time. Clayey silt soils similar to these typically require approximately 6 to 8% lime by weight. The clayey silt soils have to be moisture conditioned as part of the placement and compaction of the soils. The lime treatment should be well blended with the native soils until uniformly distributed, typically, tilling equipment is used to blend the lime with the soil. The treated soils should be placed in lifts of about 9-in.-thick and compacted with several passes with a large segmented pad compactor.

Pavement Sections. We anticipate that new paving for the project will consist of asphaltic concrete (A.C.). The pavement sections assume a 20-year design life, a substantial section of good subgrade soils (no expansive soils as subgrade), construction in conformance with current Oregon Department of Transportation specifications, and the storm water detention system installed in accordance with the manufacturer's recommendations.

For new sections of A.C. pavement, we recommend 4 in. of A.C. over 8 in. of ¾-in.-minus crushed rock base course for the roadway sections and 3 in. of A.C. over 8 in. of ¾-in.-minus crushed rock base course for the parking areas. We anticipate that an additional 18 to 24 in. of overexcavation will be required to remove the majority of expansive soils from the pavement areas. The overexcavated material can be replaced with additional imported crushed rock. The aggregate base should be compacted to at least 95%

of the maximum dry density as determined by AASHTO T-99 using a moderate-sized smooth-drum vibratory compactor. The imported crushed rock should be durable, well-graded, and up to 4 in. in size. This section of imported crushed rock can also consist of the imported crushed rock section required for the storm water storage system.

Foundations. We anticipate that most new structures can be supported on standard spread footing foundations. Structure and foundation types have not been provided; however, we anticipate that the foundation loads will be relatively light, less than 4 kips/linear ft for continuous foundation loads and less than 20 kips for column loads. To minimize the risk of post-construction differential movements, the existing highly expansive, clayey silt soils should be overexcavated (removed) within a horizontal distance of 3 ft of any foundation.

Based on the results of our investigation and our understanding of the proposed Phase 1 structures, it is our opinion that foundation support for the new structures can be provided by conventional wall-type (continuous) and column spread footing foundations established on a minimum of 6 in. of structural fill (imported crushed rock) over undisturbed, non-expansive silt or sandstone.

Excavations for foundations should be completed using a backhoe or trackhoe equipped with a smooth-lip bucket. Subgrade soils disturbed during excavation for the foundations should be removed prior to placement of the crushed rock foundation support.

Foundations should also be embedded a horizontal distance of 10 ft from slopes. On a 3H:1V slope, this would require embedment of about 3.5 ft below existing grades to provide the recommended setback from the slope.

Footings should be established at a minimum depth of 18 in. below the lowest adjacent finished grade for frost heave protection.

The width of footings should not be less than 12 in.

All footing excavations should be observed by the geotechnical engineer of record prior to placement of rebar and concrete.

For foundations founded on structural fill as discussed above, we estimate that the total, long-term settlement of spread footings designed in accordance with the above recommendations and imposing a real bearing pressure of 2,000 psf will be less than ½ in. for continuous wall foundation loads of up to 4 kips/feet and less than ½ in. for column foundation loads of up to 20 kips.

For design purposes, the real bearing value refers to the total of dead load plus frequently and/or permanently applied live loads, and can be increased by one-third for the total of all loads; dead, live, and wind or seismic.

Lateral Load Resistance. Horizontal shear forces can be resisted by frictional forces developed between the base of spread footings and the underlying soil. The total frictional resistance between the footing and the soil is the normal force times the coefficient of friction between the soil and the base of the footing. We recommend an ultimate value of 0.4 for the coefficient of friction; the normal force is the sum of the vertical forces (dead load plus real live load). If additional lateral resistance is required, passive earth resistance against embedded footings or walls can be computed using a pressure based on an equivalent fluid with a unit weight of 300 pcf. This design passive earth pressure is appropriate only if granular structural fill is to be used for the backfill around footings.

Retaining Walls. We understand that retaining walls will be required as part of mass grading of the site and for some of the structures. In our opinion, concrete cantilevered and MSE (mechanically stabilized earth) retaining walls would both be appropriate for this project. Design lateral earth pressures for embedded walls depend on the type of construction, i.e., the ability of the wall to yield and whether the wall is drained. Possible conditions are: 1) a wall which is laterally supported at its base and top and therefore is unable to yield, and 2) a conventional cantilevered retaining wall that yields by tilting about its base. For design purposes, cantilevered retaining walls and MSE are typically assumed to be yielding.

For yielding retaining wall conditions, cantilevered and MSE retaining walls can be designed based on an equivalent fluid pressure of 35 pcf. The design criteria for both types of equivalent fluid pressures assume the wall will be backfilled within 2 ft of the back of the wall with relatively clean (less than 10% passing the No. 200 sieve – washed analysis) granular fill. A non-woven geotextile (minimum 5 oz weight) should be placed between any drain material and any soil classified as sand or finer. The backfill should be placed in horizontal lifts not to exceed 9 in. (loose) and compacted to about 93% of the maximum dry density as determined by ASTM D 698. Overcompaction of the backfill should be avoided, and heavy compactors and large pieces of construction equipment should not operate within 10 ft of embedded walls. Compaction within 10 ft of the walls should be accomplished using hand-operated compactors.

The above lateral earth pressures for design of the walls do not include any lateral loads from building foundations or traffic. We recommend that building foundations be located a distance away from any retaining wall that is at least as great as the height of the wall. We recommend an additional surcharge for traffic loading with a resulting uniform equivalent lateral earth pressure of 150 psf for the traffic surcharge load. We also recommend an additional lateral earth pressure of 15 pcf of the static lateral earth pressure for a seismic event. For design purposes, the resultant of the seismic force should be assumed to act at a point two-thirds from the base of the wall.

To reduce the risk of failure of the slope due to overloading from the retaining walls, the toe of any wall located above a slope should be located at least a horizontal distance of 10 ft from the face of the slope for slopes graded to 2H:1V. This will require a toe embedment of 5 ft below adjacent grades.

Rock Excavation. Final grades have not been provided but we anticipate that cuts of up to 5 ft may be required for mass grading of roadways. In addition, sections of utilities may be founded at depths of greater than 5 ft. Practical refusal of the trackhoe was encountered on sandstone at depths of 4 to 9 ft. There is significant risk of encountering hard sandstone requiring rock excavation methods in cuts of over 7 ft and in utility trenches of greater than 7 ft deep. The risks can be minimalized by limiting the depth of cuts and utility trenches. Excavation for utility trenches in hard rock can be completed using rock saws to excavate the rock. In our opinion, a dozer with a large ripper shank will likely not be adequate for excavation of harder rock at the site. Blasting should not be allowed due to the proximity of the residential areas to the rock and due to the potential for damaging the subgrade soils by blasting. The excavation spoils from the trenches are not suitable for use as aggregate base rock in roadway sections. If properly placed and compacted, rock debris less than 2 in. in size may be used for mass grading of the site.

Utility Trenches. All utility trench excavations should be backfilled with relatively clean, granular material, such as sand, sandy gravel, or crushed rock of up to 2-in. maximum size and having less than 5% passing the No. 200 sieve (washed analysis). In our opinion, ¾-in.-minus crushed rock would be suitable for this purpose. The granular backfill material should be compacted to at least 95% of the maximum dry density as determined by ASTM D 698 in the upper 5 ft of the trench and to at least 90% of this density below a depth of 5 ft.

Seismic Considerations. The seismic design recommendations in this section are based on the current International Building Code (IBC) and the State of Oregon's Structural Specialty Code Amendments. The site is underlain by less than 10 ft of silt soils over soft sandstone and siltstone. Based on our review of the IBC and the results of our subsurface explorations, we recommend a Site Class B for the site.

LIMITATIONS

This report has been prepared to aid in the evaluation of this site and to assist the project engineers in the Phase 1 portion of the proposed Eagle Point National Cemetery expansion. The scope is limited to the specific improvements and locations described herein, and our description of the project represents our understanding of the preliminary aspects of the project relevant to the design and construction of the proposed cemetery expansion.

The conclusions and recommendations submitted in this report are based on the data obtained from the subsurface explorations completed at the locations discussed in this report and from other sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged that variations in soil, rock, and groundwater conditions exist between exploration locations. This report does not reflect any variations which may occur between these explorations. The nature and extent of variation may not become evident until construction. If, during construction, subsurface conditions different from those encountered in the explorations are observed or encountered, we should be advised at once so that we can observe and review these conditions and reconsider our recommendations where necessary.

Please contact AGEGC if you have any questions or require additional information.

Sincerely,
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